



Improved buckling reduction factors for reclaimed steel members considering imperfections and stresses

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ABSTRACT

This research examines the applicability of previously developed global and local reduction factors to account for the impact of amplified geometrical imperfections and induced stresses on the buckling resistance of slender steel box section columns. In evaluating the suitability of reclaimed steel structures, conducting a resistance check is critical in determining their reusability. In the buckling resistance check, existing geometric imperfections and residual stresses play a significant role, which could stem from (i) the original manufacturing process and (ii) later deformation during its lifetime. Previous investigations studied the effect of the manufacturing process on buckling resistance but did not consider deformations arising during the lifetime. In the current study, the effect of local and global imperfections stemming from deformations at a later time is investigated. The stresses and deformations arising from applied displacements are incorporated into a subsequent analysis to determine the buckling capacity. Previously developed local and global buckling formulas derived from EN1993-1-5 and EN1993-1-1 are evaluated against GMNI analyses that account for the influence of enlarged imperfections and induced stresses. The calibration process is performed by modifying the imperfection factor α in the buckling formulas. Subsequently, the developed formulas are assessed against the interaction buckling resistance of local and global buckling behaviours. This study provides enhanced tools for accurately estimating buckling capacity, particularly in scenarios involving stress formation due to imperfections, thereby improving the accuracy and safety of the application of reclaimed structural members.

1. Introduction

In an era where sustainability is reshaping engineering practices, extending the lifespan of steel structures has become a critical concern. A key challenge in this process is accurately evaluating the buckling capacity of steel-plated structures, which can be significantly affected by previous load histories and geometrical imperfections. With their legacy of past construction practices and wear from years of service, steel structures often exhibit imperfections that exceed today's manufacturing standards [1–3]. These issues may stem from initially relaxed tolerances or the gradual impact of operational damages [2]. To ensure the continued safety and performance of these structures, it is essential to adopt advanced methods for assessing how such imperfections affect their load-bearing capacity. By doing so, it is possible to enhance the structure's resilience and lifetime. In earlier research [4], the authors developed local and global buckling reduction formulas to estimate how enlarged imperfections can reduce buckling capacity. In that research program, the imperfections were introduced by modifying the initial

geometry of the member, without considering the stresses produced by the shape modification and the potential yielding of the steel material (resulting in residual deformations). This assumption may not hold in real-world conditions where stresses can exceed yield strength and imperfections exceed those specified during the design phase due to damage, accidents, or impacts [5]. This limitation highlights the need for a more comprehensive approach to evaluating buckling capacity that accounts for the potential impact of higher stresses on structural performance.

In the design of steel structures, the Eurocode's local and global buckling formulas rely on predefined imperfection values to account for geometrical imperfections and residual stresses. However, these formulas may fall short when applied to reclaimed members that have endured years of increased stresses and imperfections caused by damages that surpass the original design assumptions, ultimately reducing their buckling capacity. Geometrical and material nonlinear analysis with imperfections (GMNIA) is a robust approach for estimating reductions in buckling capacity due to the accumulated effects of enlarged

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imperfections and the induced generated stresses. Nevertheless, conducting GMNIA can be resource-intensive, time-consuming, and demands a high level of expertise. To address these challenges, this paper proposes a method to correlate the conditions of reused members with current standardised design formulas. It aims to refine the application of existing local and global buckling formulas in the Eurocode, improving their accuracy for assessing the buckling capacity of reclaimed structures.

2. Literature review

2.1. Investigations of imperfections in existing structures

Imperfections caused by impact or other damage sources can significantly alter a structure's behaviour, leading to reduced stiffness, strength, and stability. These imperfections often lead to premature buckling, stress concentrations, and nonlinear behaviour. Several studies have investigated the influence of such damage-induced imperfections on structural performance. Bedi [6] carried out parametric studies on a simply supported beam subjected to a unit load to determine the loss of stiffness associated with the damage. A 5-degree lateral deflection was applied to the middle third of the length of the girder with a maximum deflection around the mid-span. Several deflection grades were studied up to 40 degrees. The authors proposed different curves correlating the stiffness and moment of inertia of the damaged girder to the undamaged girder. The authors found that as the initial distortion of the section increases, its moment of inertia decreases because the height of the deformed section is lower than that of the undeformed section, and therefore, larger stress is observed initiated from the same loading situation. Stull and Earls [7] constructed a finite element model of a damaged I-girder, utilising 3D scanning to determine the reserved capacity of the structure. The bridge consists of a three-span continuous I-girder, where the top flanges of the I-girders are embedded in a concrete deck, leading to full composite action. The bottom flange of the I-girder is deformed towards the structure's interior with a total horizontal deformation of 175 mm. Two load configurations were analysed: the end span load configuration to develop compression in the bottom flange acting as initial imperfection and the three-span load configuration to develop tension in the bottom flange. Load-proportionality factors against the vertical displacements were plotted to compare the damaged and undamaged cases. A noticeable variation in behaviour was detected between the damaged and undamaged sections in the bridge's middle span.

Yamaguchi et al. [8] investigated a girder of an overpass bridge damaged by the collision of a truck. Finite element analysis was used to study the bending behaviour of the damaged girder, which exhibited a horizontal displacement of $1/156$ of the span length. The authors investigated three scenarios: the undamaged girder, the damaged girder with a horizontal displacement of $1/156$ and an additional magnified case of $1/104$ of the span length. It was found that the reduction in the initial stiffness compared to the undamaged girder is equal to 14 % and 31 %, respectively. The authors concluded, the maximum reduction in load capacity is only 0.8 % and 1.3 %, respectively. The reduction in capacity was attributed to the fact that the damaged girders tend to deform differently, thus changing their deformation characteristics. Wang et al. [9] performed an extensive research to investigate the behaviour of damaged steel girders. The authors performed strain measurements using different techniques to compare the behaviour of damaged and undamaged girders. The measured strains were used to validate numerical models, which were used to perform a parametric study to investigate the effect of location and shape on the carrying capacity. It was concluded that the damage reduces the stiffness of the girder. The greatest loss of stiffness occurs when the damage is located at the midspan, with increased length and rotation contributing to a more significant reduction in stiffness. Furthermore, the even distribution of plastic deformation over longer spans enhances the girder's stiffness

when damage is situated at the midspan. Nakayama and Kimura [10] investigated the capacity of three girders with varied initial imperfections, using maximum measurements obtained from actual bridges damaged by collisions. It was observed that the reduction in the capacity is small, and the collisions might increase the buckling capacity due to strain-hardening. Ikeda et al. [5] investigated the effect of collision damage patterns on the carrying capacity of riveted steel girders. Two successive load steps were applied to cause the deformation and apply the vertical load. It was found that for locally warped lower flange with small deformation, the reduction in the load-carrying capacity is small. If the out-of-plane deformation is larger than $1/200$, the load carrying capacity and the stiffness are significantly reduced.

In summary, previous research results showed that damage significantly impacts the behaviour of existing steel girders, with variations in stiffness and capacity between damaged and undamaged members. Consequently, further investigation is necessary to guide designers in determining the reduction in buckling resistance caused by increased imperfections resulting from prior damage to the structure. To address this objective, the authors have studied the stability behaviour of welded box-section columns with enlarged imperfections, the findings of which are presented in this paper.

2.2. Executed research strategy

In an earlier study [4], the authors conducted a numerical parametric analysis to examine the influence of enlarged imperfections on structural members. However, that study assumed that the reclaimed columns had not been subjected to stresses exceeding the yield point, thereby excluding any plastic deformations and considering the effect of prior load history negligible. The current study aims to explore additional scenarios involving reclaimed members that have encountered imperfections caused by damage or external forces that induced deformations and stresses in the structural member. The study utilises a well-validated numerical model to examine the impact of increased imperfection on the buckling capacity. The following imperfections are utilised in the numerical model: a global imperfection value of $L/1000$ is utilised with residual stress defined according to ECCS [11]. This imperfection provides global buckling resistances corresponding to the buckling curve "b" of EN1993-1-1 [12], as shown by several research studies [13,14]. Local imperfections are defined based on calibrations performed by the authors [15]. These imperfections are included in a numerical model to calculate resistance based on the local buckling curve in Annex B of EN1993-1-5 (Eq. (B.1) [16]). A maximum manufacturing tolerance of $b/125$ was applied, as detailed in EN1090-2: 2008+ A1: 2011 [17]. These imperfections are referred to as non-amplified imperfections and are used to calculate the reference resistance. The resistance obtained using the non-amplified imperfections is referred to as the base resistance and is represented by R_{en_base} . To assess the effects of amplifying imperfections, the non-amplified imperfections are increased by factors of 1.2, 1.5, 2.0, and 3.0. These factors are selected based on findings from several studies on imperfections in reclaimed members, which reported that measured imperfections exceeded design values by factors of 1.2 and 1.5 [2,3]. Additional imperfection levels are also considered to evaluate the influence of larger-than-expected imperfections on the structural behaviour and buckling resistance, representing worst-case scenarios. The study of larger imperfections is important because the ultimate goal is to develop solutions suitable for analytical-based hand calculation design methods. Consequently, the authors aimed to provide solutions with an expanded application range, ensuring broader practical relevance. The amplified imperfections are introduced as displacements that replicate the distribution of the original imperfections, creating new imperfections at the same locations as the non-amplified ones. The amplified imperfections and the resulting increased stresses are incorporated into the buckling analysis to simulate the impact of external damage that reused members might experience over their operational lifespan. The resistance

obtained using this method is referred to as the amplified resistance and is represented by R_{en_amp} . The analytical resistance for global buckling is determined according to the Ayrton-Perry formula specified in EN1993-1-1 [12], while the pure local buckling resistance is determined using the buckling curve specified in Annex B of EN1993-1-5 [16]. This buckling curve is proved to be the most reliable for welded box section columns [15,18]. The resulting resistance values are designated as R_{a_base} . To account for the influence of amplified imperfections and stresses, the Eurocode 3 formulas are adjusted through a calibration process. This is achieved by developing an imperfection factor α that is a function of the amplified imperfections. The calibrated analytical resistance to the numerical results is denoted as R_{a_amp} . The obtained values of α for pure local and global buckling will then be used to determine the interaction buckling resistance. To increase clarity, Table 1 summarizes the applied imperfections and residual stresses for each combinations.

The study program is performed according to the following steps:

1. Numerical parametric study using non-amplified imperfections and residual stresses.
2. Numerical parametric study using applied displacement loads corresponding to the following amplified imperfection factors: 1.2, 1.5, 2.0, and 3.0 for pure local and global buckling separately.
3. The ratio of amplified to non-amplified ($\frac{R_{en_amp}}{R_{en_base}}$) buckling resistances are determined for the pure local and global buckling cases, separately.
4. The applicability of the previously developed buckling formulas for amplified imperfections [4] is evaluated by comparing the resistances predicted by these formulas to the resistances estimated by the numerical analysis.
5. Calibration is performed if the previously developed formulas do not apply, where the analytical formulas are calibrated against the numerical results.
6. The applicability of the formulas is examined to determine the interaction (combined local and global) buckling capacity.

3. Numerical model development and verification

3.1. Numerical model development

A numerical model is constructed within the Ansys finite element software [19], employing four-node thin shell elements (SHELL181), depicted in Fig. 1. Load and boundary conditions are applied using two master nodes at the centre of the end cross-sections.

Using coupling constraints, the master nodes are connected to the nodes at the end cross-sections, creating coupling across all six degrees of freedom. The boundary conditions entail translation constraints in the global directions (UX, UY, UZ) and rotation about the longitudinal axis (RZ) for the first master node. Concurrently, the second master node is

restricted from translating in (UX, UY) and rotating in (RZ).

To determine an optimal mesh size that balances accuracy and computational efficiency, a mesh sensitivity analysis is conducted. This analysis is carried out for the smallest and largest plate widths of the box sections under investigation. The study revealed that a mesh size of $b/20$ provides a reliable estimate of the buckling resistance while maintaining computational efficiency. The details of the mesh sensitivity analysis can be found in [4,15].

Incorporating the non-amplified local and global imperfections involves manually modifying the ideal geometry of the model. This is accomplished by modifying the node positions by adding or subtracting the imperfection amplitude, simulating the inherent imperfections present in the member prior to use. Further elaboration on this modelling technique can be found in a previous study conducted by the authors [20]. Local imperfections are represented as integer multiples of half sin-waves, corresponding to the L/b ratio, where L denotes the length and b the width of the plate for box section columns. The local imperfections are modelled as half sin-waves applied to each plate along the longitudinal axis, with adjacent sides exhibiting opposing amplitudes, as demonstrated in Fig. 1.a. In contrast, global imperfections are modelled as a continuous half sin-wave spanning the entire column length, as illustrated in Fig. 1.b. Both local and global imperfections are applied for interaction buckling, as shown in Fig. 1.c. Multiple studies have demonstrated that the application of imperfections following this approach yields a robust and consistent estimation of the buckling resistance of structural members [21].

The additional amplified imperfections are applied as displacement loads distributed along the plates of the columns, according to Eq. (1) for global imperfections and Eq. (2) for local imperfections, as shown in Fig. 2. This method involves a three-step numerical analysis process: In the first step, the analysis is performed with the non-amplified imperfections and the residual stresses. In the second step, displacement loads are used to apply the amplified imperfections and induce stresses, and in the third step, the displacement loads are removed, and the compressive load is applied to the master node to perform the buckling analysis. Following each analysis, the stresses and geometry are updated using the 'INISTATE' and 'UPGEOM' commands. This step ensures that the results from the previous analysis are accurately reflected in the subsequent analysis. It should be noted that removing displacement loads during the analysis can cause slight variations in the deformed shape, known as the "spring-back effect." The geometries of the analysed columns were checked for each model, and the impact of spring-back was evaluated. This effect is highly nonlinear because the applied displacements alter the original residual stress pattern, leading to residual deformations. In tension zones, residual stresses had already reached the yield strength, which was further modified by the applied displacements. Subsequently, at the start of the third analysis step, a compressive force is applied, counteracting the spring-back effect. As a result, the buckling capacity obtained accounts for enlarged imperfections and yields a conservative, reduced buckling capacity. It is also worth mentioning that the applied displacements have the same distribution and sign as the non-amplified imperfections, leading to increased imperfections. For interaction buckling, the applied local and global imperfections are summed and applied simultaneously using the same displacement loads.

$$U_{G,Y} = U_{amp, glob} \cdot \sin\left(\frac{\pi \cdot (i-1)}{N_c}\right) \quad (1)$$

$$U_{L,Y}^F = U_{amp, loc} \cdot \sin\left(\frac{\pi \cdot N_w \cdot (i-1)}{N_c}\right) \cdot \sin\left(\frac{\pi}{b} \cdot \frac{b}{2 \cdot N_x} \cdot j\right) \quad (2)$$

where $U_{amp, loc}$ represents the amplitude of the additional local imperfection applied as displacements, whereas $U_{amp, glob}$ denotes the amplitude of the additional global imperfection, also applied as displacements. N_c refers to the number of cross-sections along the length used to model the various half sine waves, while N_w indicates the

Table 1

Summary of the combination of imperfections and residual stresses.

Performed study	Local imperfections	Global imperfections	Compressive residual stress
Non-amplified imperfections and residual stresses (R_{en_base})	Imperfections calibrated to the Annex B buckling curve of EN1993-1-5 [16] with a maximum value of the manufacturing tolerance of $b/125$.	$L/1000$	0.13fy
Amplified imperfections with increased residual stress (R_{en_amp})	$r_{amp} = 1.2, 1.5, 2.0$, and 3.0	$r_{amp} = 1.2, 1.5, 2.0$, and 3.0	0.13fy

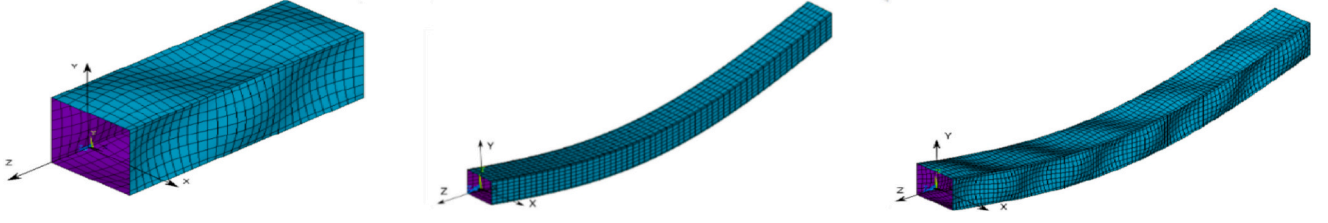


Fig. 1. a) Local, b) global, and c) interaction definitions of imperfections for box sections [20].

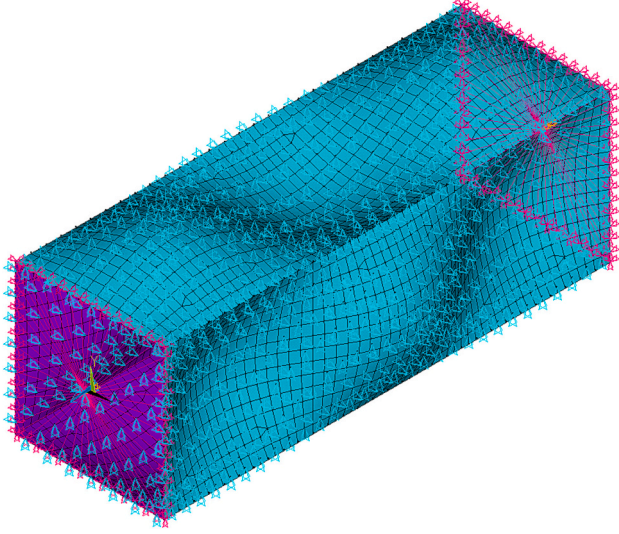


Fig. 2. Applied displacement loads to generate the amplified imperfections.

number of half sine waves along the column's length (equivalent to the length L divided by the width b). The integer i iterates from 1 to $N_c + 1$, and N_x denotes the number of points along the cross-section, with j iterating from 1 to $N_x/2$.

3.2. Applied material model

This analysis adopts a quad-linear material model to effectively capture the complex nonlinear behaviour of normal strength steel (NSS), in line with the specifications detailed in prEN 1993-1-14 [22]. The material model is visually depicted in Fig. 3 and is defined by Eqs. (3)–(8), based on the research by Yun and Gardner [23]. Extensive validation has been carried out using a comprehensive database of coupon tests to confirm the model's accuracy.

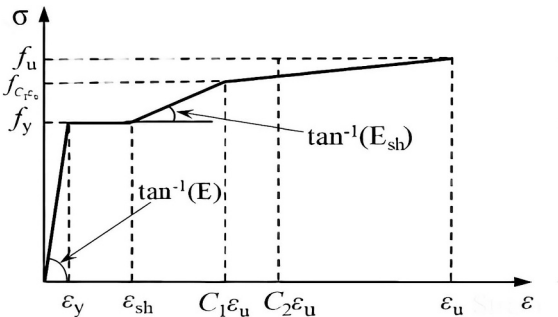


Fig. 3. Material model for Normal Strength Steel (NSS), according to prEN 1993-1-14 [22].

$$\sigma(\epsilon) = \begin{cases} E\epsilon, & \epsilon \leq \epsilon_y \\ f_y, & \epsilon_y < \epsilon \leq \epsilon_{sh} \\ f_y + E_{sh}(\epsilon - \epsilon_{sh}), & \epsilon_{sh} < \epsilon \leq C_1 \epsilon_u \\ f_y C_1 \epsilon_u + \frac{f_u - f_y C_1 \epsilon_u}{(\epsilon_u - C_1 \epsilon_u)} (\epsilon - C_1 \epsilon_u), & C_1 \epsilon_u < \epsilon \leq \epsilon_u \end{cases} \quad (3)$$

$$E_{sh} = \frac{f_u - f_y}{C_2 \epsilon_u - \epsilon_{sh}} \quad (4)$$

$$\epsilon_{sh} = 0.1 \frac{f_y}{f_u} - 0.055 \text{ but } 0.015 \leq \epsilon_{sh} \leq 0.03 \quad (5)$$

$$\epsilon_u = 0.6 \left(1 - \frac{f_y}{f_u} \right), \text{ but } 0.06 \leq \epsilon_u < A \quad (6)$$

$$C_1 = \frac{\epsilon_{sh} + 0.25(\epsilon_u - \epsilon_{sh})}{\epsilon_u} \quad (7)$$

$$C_2 = \frac{\epsilon_{sh} + 0.4(\epsilon_u - \epsilon_{sh})}{\epsilon_u} \quad (8)$$

Implementing the quad-linear model in Ansys necessitates the calculation of specific coefficients tailored to the steel grades being analysed. These coefficients are essential for defining the material behaviour and include parameters such as the yield strain ($\epsilon_y = f_y/E$), the strain hardening modulus E_{sh} , the elongation coefficient C_1 (set to 0.2 to account for post-fracture elongation), the cut-off strain coefficient C_2 (to avoid overestimation of strength), and C_2 , which determines the slope of strain hardening E_{sh} . A summary of these coefficients is provided in Table 2.

3.3. The applied residual stress model

In slender plates, geometric imperfections typically dominate the influence on buckling capacity, overshadowing the effects of residual stresses. Nonetheless, residual stresses are still significant, as they can induce premature yielding and degradation of stiffness. Hence, selecting an appropriate residual stress model is critical for accurate buckling capacity predictions. The residual stress model shown in Fig. 4 is employed in this study, as endorsed by the ECCS (European Convention for Constructional Steelworks) [11] and prEN1993-1-14 [22]. Positive values represent tensile residual stresses, whereas negative values represent compressive stresses. Additional information regarding this model is available in [15].

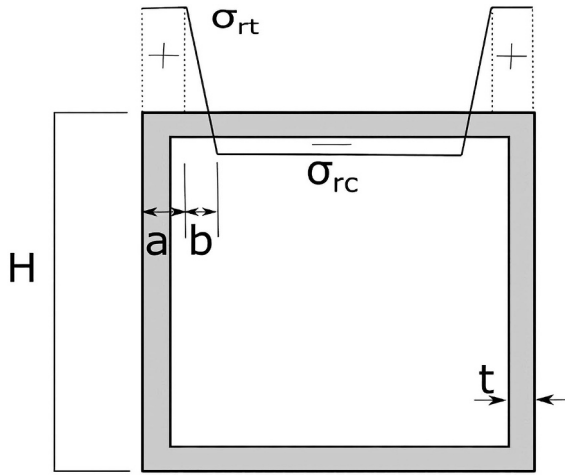
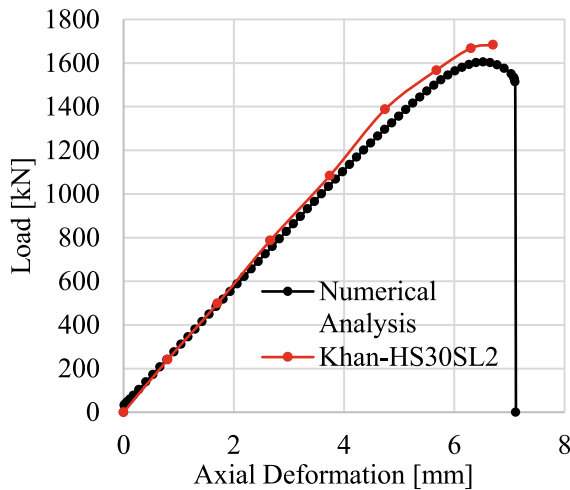
3.4. Validation of the numerical model

The validation process for the numerical models has been detailed in earlier studies conducted by the authors for box section columns [20,24]. This validation entailed comparing numerical results with experimental observations. The numerical model demonstrated a high accuracy in estimating the buckling capacity of welded box section columns, achieving a mean value of 1.0 and a coefficient of variation (CoV) of 0.06. Example of this validation is presented in Fig. 5, which

Table 2

Parameters of the applied material model.

	f_y [MPa]	f_u [MPa]	ϵ_{sh} [–]	ϵ_u [–]	$C1$ [–]	$C2$ [–]	E_{sh} [MPa]	$C1 \cdot \epsilon_u$ [–]	f_{CIcu} [MPa]
S355	355	510	0.015	0.182	0.310	0.448	2310	0.057	451

**Fig. 4.** Residual stress model for welded box section columns [11,22].**Fig. 5.** Comparison of the test and the numerical model load-displacement curves for box section column S700–150–4.92–2512 [25].

demonstrates load-deformation curves from both experimental and numerical analyses.

The sample shown here is obtained from a research program by Khan et al. [25]. Fig. 6 illustrates the failure mode observed at the final load step of the buckling tests, illustrating the actual experimental results, total deformations, and Von-Mises stresses for both columns. It is evident that both models effectively replicate the buckling behaviour observed during the experimental tests.

4. Investigation of the buckling behaviour of box section columns

4.1. Parameters used in the numerical study

This Section outlines the parameters used in the parametric study of box section columns to examine local, global, and interaction buckling

behaviour. To investigate pure local buckling behaviour, the selected cross-sections were designed with a length three times the width, with a width (b) over thickness (t) ratio ensuring a plate local slenderness ratio ($\bar{\lambda}_p$) greater than 0.7. In contrast, examining pure global buckling necessitates cross-sections with a plate local slenderness ratio ($\bar{\lambda}_p$) below 0.7. To investigate the interaction buckling behaviour, a broad spectrum of slenderness ratios was examined: the local slenderness ratio ($\bar{\lambda}_p$) ranged from 0.7 to 2.0, while the global slenderness ratio ($\bar{\lambda}_g$) varied between 0.2 and 2.0. Plate widths were between 200 mm and 450 mm, and thicknesses ranged from 3 mm to 12 mm, yielding a wide array of local slenderness ratios. Column lengths were systematically adjusted from 750 mm to 20,000 mm, generating a comprehensive range of global slenderness ratios. Detailed characteristics of the columns examined are presented in Table 3.

The numerical study investigated (i) pure local buckling, (ii) pure global buckling, and (iii) interaction buckling. A total of 400 GMNI analyses are conducted within the current parametric study.

4.2. Description of the performed study – Demonstrative example

The study examines the imperfections that develop during the service life of reclaimed members, where external forces induce stresses and imperfections within the plates of these members. The process of applying local and global imperfections unfolds in three distinct steps, as described in Section 3.1. The main objective is to illustrate how varying the magnitude of imperfections and the generated stresses affect the numerical capacity for pure global, local, and interaction buckling behaviours. Non-amplified imperfections and residual stresses, as outlined in Section 3.3, are incorporated into the numerical model. These imperfections yield estimations of buckling resistance that closely match the Annex B curve and the Ayrton-Perry formula, with α values of 0.34 for local and global buckling, respectively. The magnitudes of these imperfections were then increased using several amplification factors (r_{amp}), as shown in Table 1. An example of the performed parametric study is shown in Fig. 7, where the y-axis shows the global buckling resistance, the x-axis shows the global slenderness ratio ($\bar{\lambda}_g$). The numerical model incorporates a non-amplified imperfection of $L/1000$, implemented by adjusting the node positions. The residual stresses are applied according to Section 3.3. The resistance derived from this combination is a benchmark for comparing with other amplified imperfections. Additional amplified imperfections are also examined, including $L/833$, $L/666$, $L/500$, and $L/333$. These values correspond to the amplification ratios (r_{amp}), illustrated in Fig. 7. The amplified imperfection, larger than $L/1000$, are applied as displacement loads, as described previously. The results of the performed study show that the most significant reduction in capacity is observed when the applied imperfections are increased to three times the non-amplified level.

To calibrate the analytical results to the numerical results, the resistances obtained with amplified imperfections are normalised. This is achieved by dividing the resistance obtained with the amplified imperfections ($R_{en,amp}$) by the non-amplified resistance ($R_{en,base}$) obtained using the non-amplified imperfections. This normalisation facilitates a comparative analysis, highlighting the impact of imperfection magnitude and the induced stresses on buckling resistance. In the Eurocode, the α factor accounts for imperfections and residual stress in the buckling curves. Adjusting the value of α allows the analytical design formulas to be calibrated against the amplified numerical resistances, replicating the trend achieved by the numerical results. The subsequent

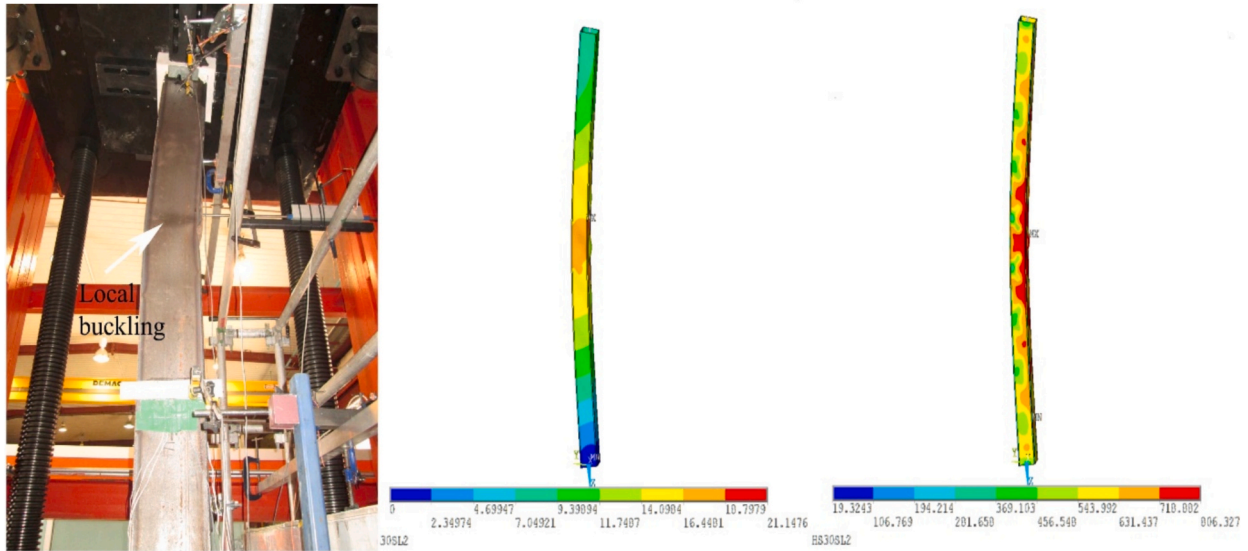


Fig. 6. The numerically obtained failure mode for interaction buckling at the final loading step for Khan et al. test specimen S700–150–4.92–2512 [25]; a) experimental test [25], b) deformed shape, c) Von-Mises stresses.

Table 3

Geometrical properties of the analysed cross-sections.

Property [unit]	Range (box section)
Global slenderness ratio $\bar{\lambda}_g$ [–]	0.2–2.0
Local slenderness ratio $\bar{\lambda}_{pf}$ [–]	0.7–2.2
Local slenderness ratio $\bar{\lambda}_{pw}$ [–]	0.7–2.2
flange thickness t_f [mm]	3–6
Web thickness t_w [mm]	3–6
Plate height h [mm]	250
Plate width b [mm]	250–500
Plate length L [mm]	750–20,000
Steel yield strength f_y [MPa]	355

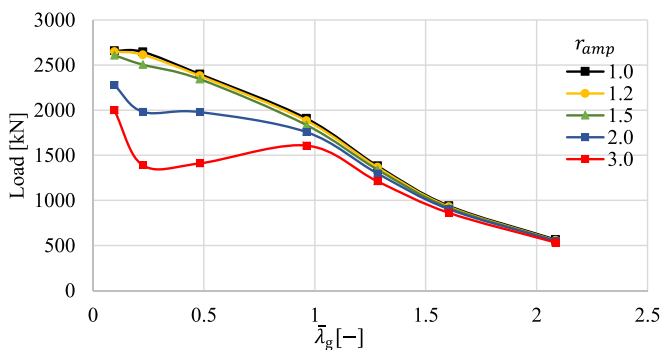


Fig. 7. Performed parametric study for pure global buckling for different values of imperfections using the applied displacement loads.

Sections of this study will demonstrate the calibration process in detail.

4.3. Displacement load imperfections method

4.3.1. Local buckling resistance

Short columns with lengths three times the width are analysed to investigate the local buckling resistance, as detailed in Section 4.1. In a prior investigation [15], the authors developed local imperfections that are calibrated to the buckling curve of Annex B of EN 1993-1-5, as described in Section 2.2. These imperfections are regarded as non-amplified imperfections, and the numerical resistance obtained using

these imperfections is considered the non-amplified buckling capacity, denoted as R_{en_base} . These imperfections are amplified using amplification ratios (r_{amp}) shown in Table 1. The resistance obtained using these imperfections is termed the amplified buckling resistance, denoted as R_{en_amp} . To investigate the effect of amplifying the imperfections, Fig. 8 shows the load-deformation curves for three samples with the following local slenderness ratio ($\bar{\lambda}_p$), 0.7, 1.35 and 2.16. For each sample, the figure compares the behaviour of the sample with non-amplified imperfections (No-amp) against the behaviour of amplified imperfections ($r_{amp} = 3.0$). The amplified imperfections are applied using two methods: (i) through modifications of node positions (Node-amp), representing the previous research conducted by the authors [4], and (ii) through displacement loads (Disp-amp). The figure displays the Von Mises stresses for the Disp-amp method at the shown points along the load-deformation curve. The first image on the left presents the Von Mises stress at the beginning of the buckling analysis. The Node-amp method assumes that the imperfections do not induce additional stresses when they occur during the member's lifetime. In contrast, the Disp-amp method accounts for the effects of amplified imperfections combined with the stress generation during the formation of these imperfections. For samples with small local slenderness ($\bar{\lambda}_p \approx 0.7$), the difference between the two methods is negligible, with both methods producing identical load-deformation curves and reductions in resistance compared to the non-amplified imperfection resistance. In the medium slenderness range ($\bar{\lambda}_p \approx 1.35$), the Disp-amp method yields a lower buckling resistance than the Node-amp method. This is due to the influence of local imperfections in this range. However, due to strain hardening, a larger stiffness is observed, which is consistent with findings from several researchers, as detailed in the literature review [9,10]. In the large slenderness range ($\bar{\lambda}_p \approx 2.16$), the ultimate buckling capacity of the new method is slightly lower. However, the Disp-amp method exhibits greater stiffness up to the ultimate capacity due to strain hardening. After reaching the ultimate capacity, the residual capacity of the Disp-amp method is lower compared to the Node-amp method.

The results of the performed parametric study are shown in Fig. 9.a, where the x-axis shows the local slenderness ratio ($\bar{\lambda}_p$) and the y-axis shows the normalised amplified buckling resistance (R_{en_amp}/R_{en_base}). It can be observed, the obtained trend is slightly different from the behaviour observed in the previous investigations. When the imperfections are amplified threefold, the maximum reduction in buckling

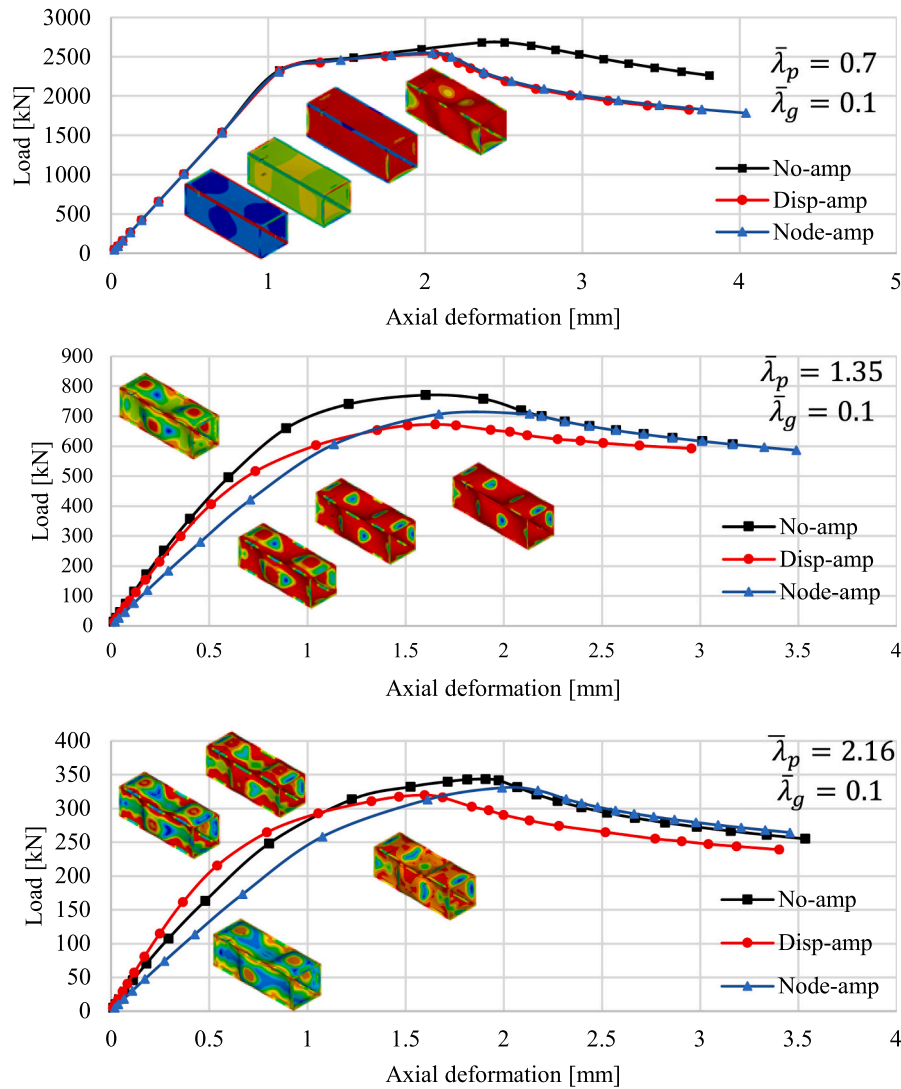


Fig. 8. Load-deformation curves for pure local buckling behaviour for three samples with the following local slenderness ratios ($\bar{\lambda}_p$) of 0.7, 1.35, 2.16.

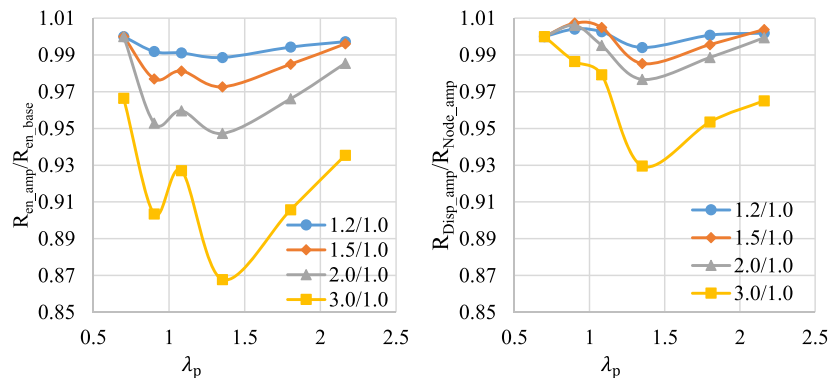


Fig. 9. a) Normalised numerical results for pure local buckling analysis of box section columns using displacement load imperfection method (Disp-amp), b) comparison between the Disp-amp and the Node-amp methods for pure local buckling.

capacity reaches 12 %, contrasting with the previous method, where the maximum reduction was only 9 %. This is due to the fact that the applied displacements induce larger stresses in the plates of the member, leaving a smaller reserved capacity. Particularly for sections characterised by medium local plate slenderness, where the plates have the largest sensitivity to local imperfections, as shown previously by the authors

[15]. The maximum difference between the two methods is equal to 7 % and occurs in the medium slenderness range. The calibration process involves adjusting the imperfection factor (α) in the Annex B buckling curve (Eq. (B.1) [16]) to match the normalised amplified buckling resistance (R_{en_amp}/R_{en_base}). Given that the results are similar to those from previous analyses [4], the applicability of the previously developed

formula (Eq. (9)) for the imperfection factor (α) is evaluated using the new results obtained with the Disp-amp method. The result of the previous calibration is shown in Fig. 9.a in [4], where the normalised amplified analytical resistance is shown for each amplification ratio. Due to the mathematical nature of the buckling curve of Annex B EN 1993-1-5, slightly different trends are obtained for each amplification ratio. Fig. 10 shows a graphical representation of Eq. (9), where the x-axis shows the amplification factor (r_{amp}) and the y-axis shows the corresponding imperfection factor (α). Fig. 9.b shows a comparison between the Disp-amp and Node-amp methods. It can be seen that the Disp-amp method yields smaller resistances than the Node-amp method for the medium slenderness range $\bar{\lambda}_g \approx 1 - 1.5$.

The ability of the provided formula to yield buckling resistances close to the ones predicted by the numerical analyses is shown in Table 4. It demonstrates the statistical measures of the ratio of the normalised numerical resistance to the normalised calibrated resistance for the investigated amplification factors. It is observed that the mean value for amplification ratios of 2.0 or less is equal to 1.0, indicating safe results. However, for $r_{amp} = 3.0$, it is equal to 0.97. This discrepancy can be because significantly large imperfections can induce the yielding of the plates, while smaller imperfections do not progress to the yielding phase and, therefore, do not reduce the capacity of the plates significantly. The coefficient of variation for all the amplification ratios is small, indicating less scattering between the numerical and the calibrated analytical resistances. Therefore, according to this method, the previously developed formula for the imperfection factor (α) can be used to estimate the buckling capacity of columns that experienced imperfections caused by external forces that led to generating stresses within the member. The developed formula can be used in the Annex B buckling formula, as shown by Eq. (10).

$$\alpha_L = 0.34 + 0.065 \cdot (r_{amp} - 1) \quad (9)$$

$$\rho = \frac{1}{\phi_p + \sqrt{\phi_p^2 - \bar{\lambda}_p^2}} \quad (10)$$

$$\phi_p = \frac{1}{2} \left(1 + (0.34 + 0.065 \cdot (r_{amp} - 1)) (\bar{\lambda}_p - 0.7) + \bar{\lambda}_p \right)$$

4.3.2. Global buckling resistance

The pure global buckling behaviour is investigated by applying the geometries described in Section 4.1. The non-amplified imperfection is applied by modifying the position of the nodes of the column, while the amplified imperfection is applied as a displacement along the entire length of the column, according to Eq. (1). A global imperfection of $L/1000$, combined with the residual stress model described in Section 3.3 are used. Several researchers demonstrated that this method provides a close estimation of the buckling capacity of the Eurocode buckling curve 'b' [13]. These imperfections are considered non-amplified imperfections, and the buckling capacity obtained by applying these imperfections is considered the non-amplified buckling capacity, denoted as R_{en_base} . The non-amplified imperfections are then amplified using amplification ratios (r_{amp}). The resistance obtained using the amplified

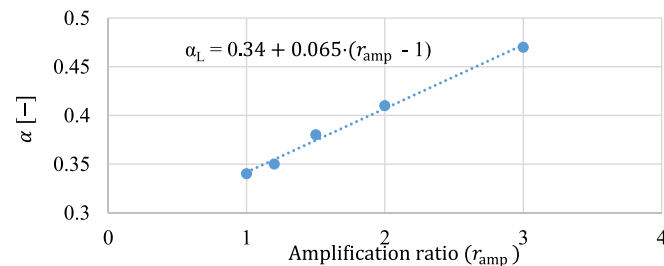


Fig. 10. The relationship between the amplification ratio (r_{amp}) and the α value in Eq. (9) [16].

Table 4

Statistical measures evaluating the ratio of the normalised numerical resistances to the normalised calibrated Eurocode resistances for pure local buckling of box section columns for the Disp-amp method.

Statistical measures	Imperfection amplification ratio (r_{amp})			
	1.20/1.0	1.5/1.0	2.0/1.0	3.0/1.0
Max	1.01	1.02	1.03	1.01
Min	1.00	0.99	0.98	0.93
μ	1.00	1.00	1.00	0.97
σ	0.00	0.01	0.02	0.03
CoV	0.00	0.01	0.02	0.03

imperfections is considered the amplified buckling capacity, denoted as R_{en_amp} . Similar to the comparison for local buckling behaviour, a comparative analysis is conducted for global buckling behaviour. This involves comparing the method used in the previous study [4], which applies amplified imperfections by modifying node positions (Node-amp), with the current method that applies imperfections as displacement loads (Disp-amp). The comparison is shown in Fig. 11, where the load-displacement curve is shown for the non-amplified imperfection (No-amp), the Node-amp and Disp-amp methods for three samples with three different global slenderness ratios ($\bar{\lambda}_g$), namely, 0.22, 1.28, 2.1. The Node-amp and Disp-amp methods, both with an amplification factor ($r_{amp} = 3.0$). The figure displays the Von Mises stresses for the Disp-amp method at the shown points along the load-deformation curve. The first image on the left presents the Von Mises stress at the beginning of the buckling analysis. It can be observed, for the small global slenderness range ($\bar{\lambda}_g \approx 0.22$), the reduction in the buckling capacity is very large compared to the Node-amp method where the generated stresses due to the applied imperfection lead to yielding of the plates and therefore reducing the buckling capacity, as shown by Fig. 11.a. For the medium global slenderness range ($\bar{\lambda}_g \approx 1.28$), the Disp-amp method shows a larger stiffness with slightly larger buckling capacity compared to the Node-amp due to the strain hardening effect that occurred during the application of the displacement loads, as shown by Fig. 11.b. For columns with large slenderness ratio ($\bar{\lambda}_g \approx 2.1$), the effect of the strain hardening intensifies leading to a relatively larger stiffness and therefore larger buckling capacity, as shown by Fig. 11.c.

The result of the performed study is shown in Fig. 12.a, where the x-axis shows the global slenderness ratio ($\bar{\lambda}_g$), the y-axis shows the normalised numerical resistance (R_{en_amp}/R_{en_base}) for each amplification ratio (r_{amp}). The findings indicate that the reduction in the buckling capacity is notably greater compared to the reduction observed when the imperfections were increased without considering the generated stresses, as demonstrated in the previous study [4], particularly when amplified to three times the non-amplified imperfections. This difference arises because larger imperfections applied as displacements cause the central section of the column to incur large stresses that can reach yielding, whereas smaller imperfections keep the column below the yield point. Consequently, when a compressive load is applied in the subsequent analysis, the column's larger yielded areas have a smaller reserve capacity. This is more significant in shorter columns, where the stiffness of the column is larger and, therefore, the applied imperfections lead to larger stresses in the middle region. It is possible to see that the maximum reduction in the buckling capacity occurs in the region of the small slenderness range, $\bar{\lambda}_g \approx 0.2 - 1.0$. The maximum reduction in the buckling capacity amounts to 35 %, in contrast to the 20 % reduction outlined in the previous study [4].

The validity of the previously calibrated analytical results in [4] is examined against the results of the current method. The calibration process is performed by calibrating the normalised analytical resistance obtained using the Ayrton-Perry formula of the EN 1993-1-1 (Eq. (6.49) [12]) against the normalised numerical results [4]. The numerical resistances are normalised by dividing by the non-amplified resistance

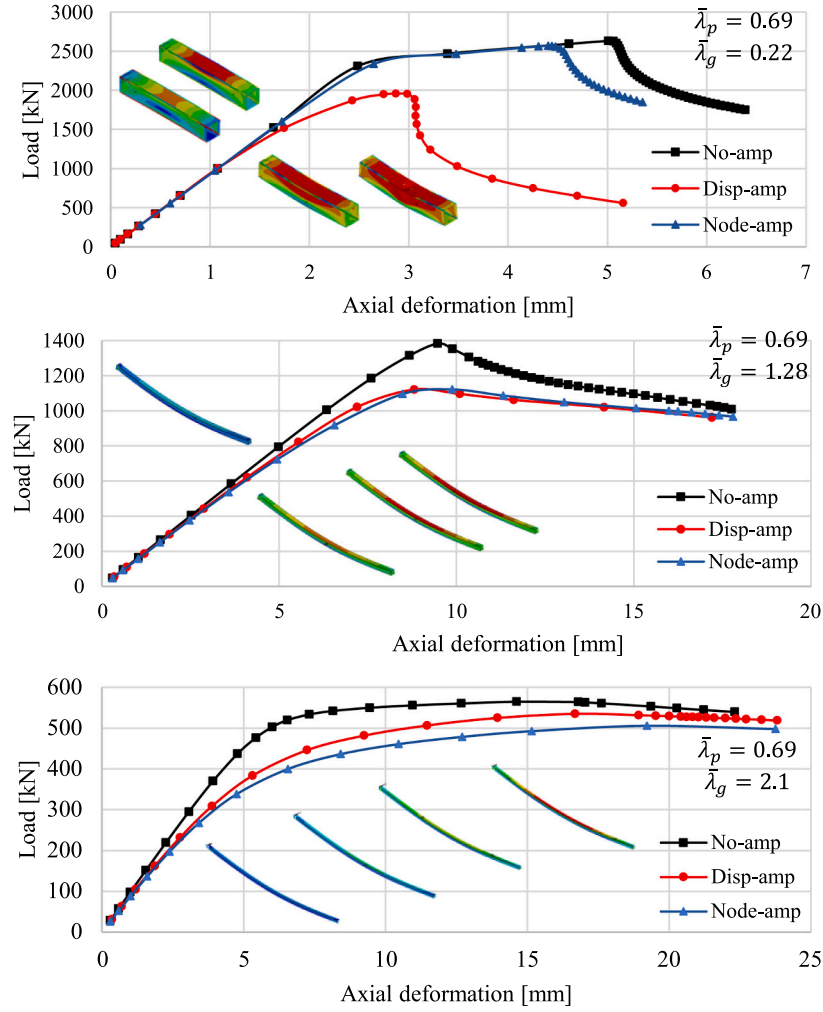


Fig. 11. Load-deformation curves for pure global buckling behaviour for three samples with the following global slenderness ratio $\bar{\lambda}_g$ of 0.22, 1.28, 2.1.

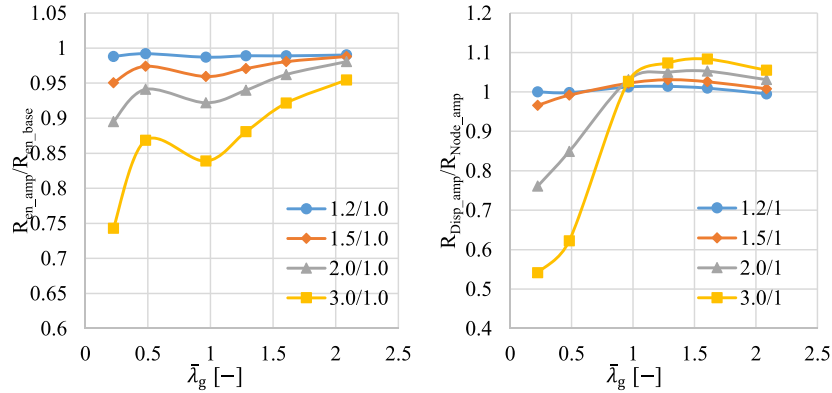


Fig. 12. a) Normalised numerical results for pure global buckling analysis of box section columns using the displacement load imperfection method (Disp-amp), b) comparison between the Disp-amp and the Node-amp methods for pure global buckling.

(R_{en_base}) for the numerical results and (R_{a_base}) for the analytical results. Fig. 12.a in [4] illustrates the outcomes of calibrating the analytical resistance to the numerical results by adjusting the imperfection factor (α) in Eq. (6.49) [12] to achieve a close estimation of the normalised numerical resistances along the global slenderness range for each amplification factor [4]. Fig. 12.b shows a comparison between the Disp-amp and Node-amp methods. It can be seen, the Disp-amp method yields smaller resistances than the Node-amp method for the slenderness range

$\bar{\lambda}_g < 1$ and larger otherwise.

Fig. 13 represents the obtained imperfection factor (α) for each amplification factor (r_{amp}). The previously developed formula for the imperfection factor (α) for r_{amp} is depicted by Eq. (11). To investigate the accuracy of the proposed formula in Eq. (11), the statistical measures of the ratio of the numerical-to-calibrated resistances are shown in Table 5. The reduction factor for the calibrated resistance is calculated according to Eq. (12). It is evident that the obtained mean value for all the

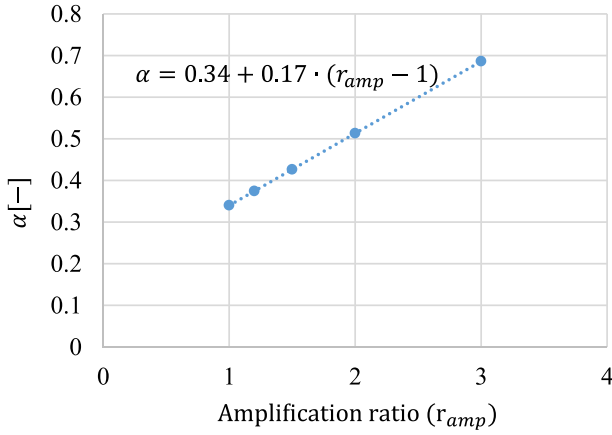


Fig. 13. Correlation between the amplification ratio (r_{amp}) and the α imperfection factor in Eq. (6.49) [12] using the displacement load imperfection method.

Table 5

Statistical measures evaluating the ratio of the normalised numerical resistances to the normalised calibrated Eurocode resistances for pure global buckling of box section columns using the displacement load imperfection method.

Statistical measures	Imperfection amplification ratio (r_{amp})			
	1.20/1.0	1.5/1.0	2.0/1.0	3.0/1.0
Max	1.01	1.03	1.05	1.10
Min	0.99	0.95	0.90	0.75
μ	1.00	1.01	1.01	1.00
σ	0.01	0.03	0.06	0.13
CoV	0.01	0.03	0.06	0.13

amplification ratios (r_{amp}) is equal to 1.0, and the coefficient of variations is relatively small for small values of $r_{amp} < 1.5$. However, it is larger for larger amplification ratios r_{amp} . This can be interpreted in relation to the studied geometries and yield strength: when the applied imperfections exceed 1.5 times the non-amplified imperfections, stress levels approach the yield strength, leading to minimal reserve capacity in the member. Due to the nature of the Ayrton-Perry formula of the Eurocode, larger discrepancy is inevitable due to the fact that the imperfections factor (α) should depend on the value of the global slenderness to achieve better agreement between the numerical and the analytical results, as was previously shown by Somodi et al. [26].

$$\alpha_g = 0.34 + 0.17 \cdot (r_{amp} - 1) \quad (11)$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_g^2}} \quad (12)$$

$$\phi = 0.5 \left[1 + \alpha_g (\bar{\lambda}_g - 0.2) + \bar{\lambda}_g^2 \right]$$

4.3.3. Interaction buckling resistance

To investigate the interaction buckling behaviour using amplified imperfections, the geometries detailed in Section 4.1 are utilised. The investigated columns are slender both locally and globally. Therefore, both local and global imperfections are to be applied. The previously discussed imperfections for pure local and pure global buckling are applied here, specifically $L/1000$ for global imperfections and those calibrated for the Annex B curve, with a maximum value of $b/125$, combined with residual stresses, as outlined in Section 3.3. These imperfections are denoted as non-amplified imperfections. The previously shown amplification factors are used to amplify the magnitude of these imperfections. The non-amplified numerical resistances obtained using local and global non-amplified imperfections are denoted as R_{en_base} .

The numerical results obtained using the amplified imperfections is considered the amplified resistance, denoted as R_{en_amp} . The amplified numerical resistance is normalised by dividing by the non-amplified resistance (R_{en_amp}/R_{en_base}). To demonstrate the difference in behaviour between the two previously discussed methods of applying the imperfections, two samples are shown in Fig. 14 that compare the load-deformation diagrams of Disp-amp method and the Node-amp method. The samples have a local slenderness ratio ($\bar{\lambda}_p$) of 1.35 and global slenderness ratios ($\bar{\lambda}_g$) of 0.50 and 1.28. For the sample with $\bar{\lambda}_g$ of 0.50, the reduction in the buckling capacity in Disp-amp method is relatively large compared to that of the Node-amp method due to the stresses generated in the plates of the column due to the application of the local and global imperfection as shown in Fig. 14.a, where the generated stresses in plates lead to a lower buckling capacity. In contrast, for the sample with a $\bar{\lambda}_g$ of 1.28, strain hardening increases the stiffness of the plates, bringing the buckling capacity closer to that of the Node-amp method by counteracting the yielding of the member's plates.

The outcomes of the parametric study are illustrated in Fig. 15, where six subplots are shown corresponding to the investigated local slenderness ratio ($\bar{\lambda}_p$). In the case of the interaction buckling, a similar behaviour to that of the pure global buckling is obtained, where the maximum reduction occurs in the low-to-medium global slenderness range $\bar{\lambda}_g \approx 0.2 - 1.0$. Nevertheless, the reduction in the buckling capacity is significantly larger than that observed for pure global buckling, indicating a further decrease in the buckling capacity when both buckling behaviours subsequently occur. The stresses generated due to the local and global imperfections significantly reduce the buckling capacity for short columns. It can be seen for the local slenderness ratio $\bar{\lambda}_p > 1.40$, the reduction is large when the amplified imperfections are three times the non-amplified imperfections. This fact results from the substantial stresses induced in the plates of short columns by the application of large global and local imperfections.

The non-amplified analytical resistance is obtained according to Eq. (6.48) outlined in EN1993-1-1 [12], where the reduction factors for local and global buckling are determined according to Eq. (6.49) of EN1993-1-1 [12] and Eq. (B.1) of EN1993-1-5 [16], with an α value of 0.34. The amplified analytical resistance is determined by employing the derived pure local and global imperfection factor (α), as per Eq. (9) and Eq. (11), incorporated into Eq. (10) and Eq. (12), respectively. The outcomes of the estimated analytical interaction buckling resistances are illustrated in Fig. 15 in [4], where the normalised analytical buckling resistance is shown. Owing to the mathematical nature of the reduction factor formulas, the obtained trend for the analytical buckling capacity differs from that observed in the numerical results. In the analytical results, the greatest reduction in buckling capacity is observed within the medium-to-large global slenderness, while for the numerical results, the maximum reduction is in the low global slenderness range, where applying both imperfections with large magnitude leads to generating substantial stresses and, therefore, reducing the capacity drastically. Fig. 16 shows a comparison between the resistances obtained by the Disp-amp method and the Node-amp method. For $\bar{\lambda}_g < 0.5$, the Disp-amp method results in lower resistances than the Node-amp method, which can be attributed to the high stresses induced in the plates of stiffer short columns due to large global and local imperfections. For $\bar{\lambda}_g > 0.5$, the Disp-amp method results in larger resistances than the Node-amp method. This is due to the strain hardening effect from the applied imperfections, which outweighs the impact of generated stresses. For columns with larger global slenderness, the stresses generated by the imperfections are less significant than in short, stiff columns. For small values of amplification, the difference between the two method is small, however for large amplification factor $r_{amp} = 3$, the difference becomes significant, potentially reaching up to twice the value for high global slenderness. Table 6 shows the statistical measures of the ratio of the normalised numerical to the normalised analytical

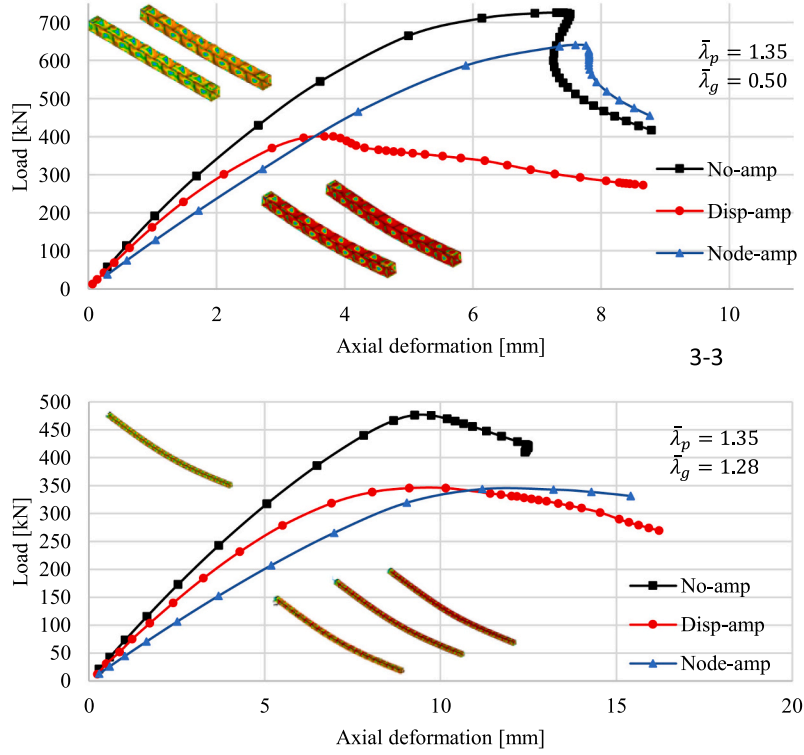


Fig. 14. Load-deformation curves for interaction buckling behaviour for two samples with a global slenderness ratio ($\bar{\lambda}_g$) of 0.5 and 1.28 and a local slenderness ratio ($\bar{\lambda}_p$) of 1.35.

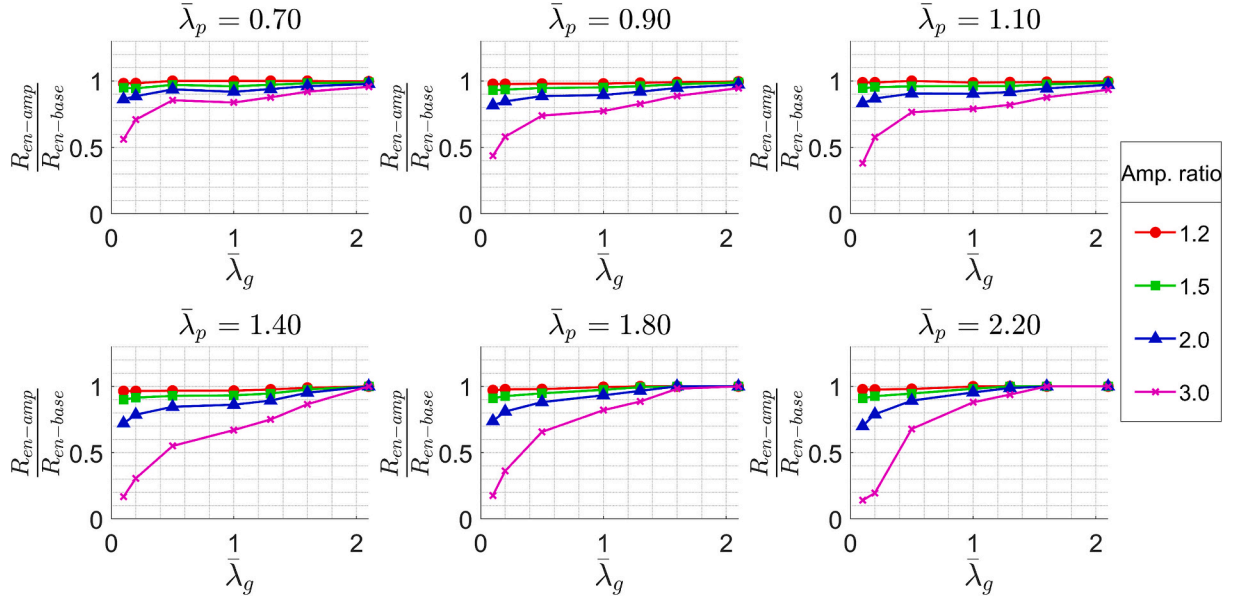


Fig. 15. Normalised numerical results for the interaction of local and global buckling behaviours of box section columns using the displacement load imperfection method (Disp-amp).

resistances. The obtained mean value for the amplification ratios less than or equal to 2.0 is larger than 1.0, indicating safe results. However, for $r_{amp} = 3$, the mean value is less than one. The coefficient of variation is relatively small for low amplification factors, whereas it becomes significantly larger for high amplification factors.

5. Conclusions

In earlier research [4], the authors developed formulas to account for the impact of amplified imperfections. However, these formulas overlooked the influence of the induced stresses during the formation of the imperfections. This study investigated the effectiveness of these formulas by examining how stresses generated during the formation of imperfections affect the buckling capacity. The global buckling formula

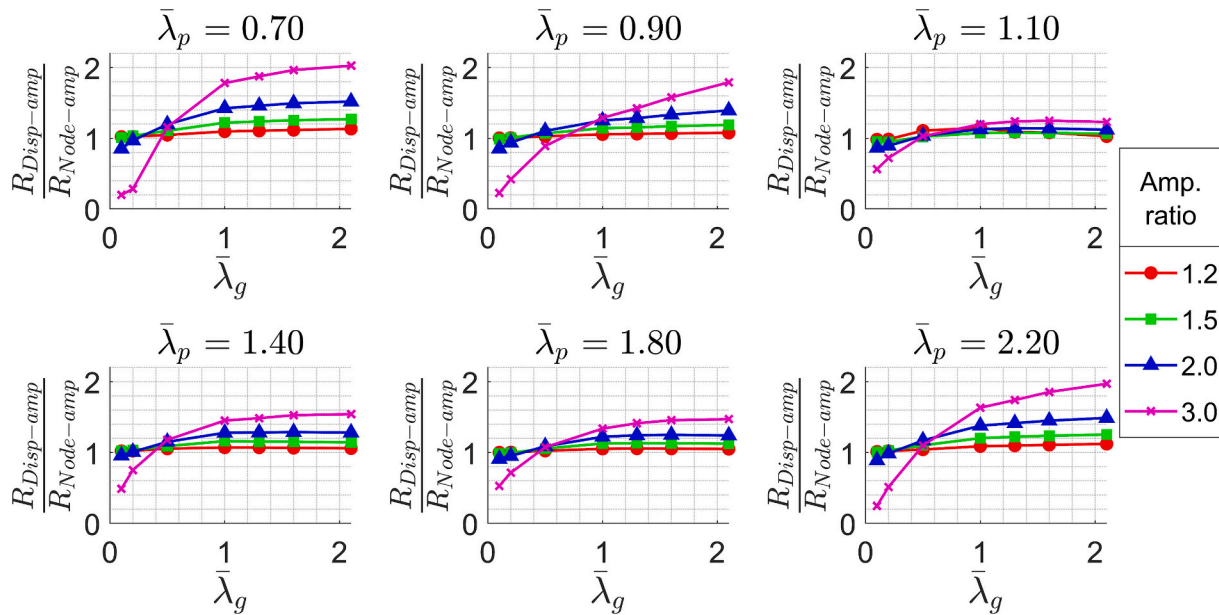


Fig. 16. Comparison between the Disp-amp and the Node-amp methods for interaction buckling.

Table 6

Statistical measures evaluating the ratio of the normalised numerical resistances to the normalised calibrated Eurocode resistances for interaction buckling of box-section columns.

Statistical measures	Imperfection amplification ratio (r_{amp})			
	1.20/1.0	1.5/1.0	2.0/1.0	3.0/1.0
Max	1.12	1.13	1.22	1.39
Min	0.97	0.93	0.82	0.22
μ	1.02	1.02	1.03	0.97
σ	0.03	0.05	0.10	0.28
CoV	0.03	0.05	0.10	0.29

outlined in EN1993-1-1 [12] and the local buckling formula of EN1993-1-5 Annex B [16] can be modified to consider the influence of the generated stresses and imperfections by adjusting the value of the imperfection factor (α). Local and global imperfections, developed to align with the mentioned buckling capacity, were utilised to obtain the non-amplified buckling resistance [15,26]. To investigate the influence of stresses developed in the member during the development of the imperfections, the amplified imperfections are applied as displacements along the entire length of the column. The resulting deformation and stresses are then incorporated into the next step of the analysis, where a compressive load is applied to determine the buckling capacity. Using this method, the reduction in the buckling capacity is more significant compared to amplifying imperfections, neglecting the generated stress. The reduction in buckling capacity in this method can reach up to 35 % of the original buckling capacity when one imperfection is tripled. This considerable decrease in buckling capacity primarily stems from generated stresses in the plates, which may reach the yield strength, resulting in minimal preserved capacity. It was found that the developed linear formulas in the previous study [4] can be used to estimate the buckling capacity for the amplified imperfection, accounting for stresses generated by amplifications up to twice the magnitude of the non-amplified imperfections, with a mean value larger than 1.0.

The developed equations for the imperfection factor (α) are compatible with the current Eurocode design provisions and are applicable to determine the local, global and interaction buckling capacity of welded box section columns if the imperfection magnitude is known. The currently developed new design proposals are well-applicable for evaluating the buckling resistance of (i) existing structures which are

planned to be used further or (ii) for steel members which are to be reused. The currently proposed design equations can significantly support the assessment and the resistance determination procedure of reused steel members. The proposed formulas can be employed by designers in a manner analogous to the existing buckling expressions in the Eurocode, offering a direct and practical means to account for enlarged imperfections in design calculations.

Future work could focus on refining and validating the newly proposed formulas for enlarged imperfections through stochastic analyses that incorporate various imperfection patterns and amplitudes. The reliability analysis should also be performed for the proposed design equations, which is a future task and beyond the aim of the current paper. Additionally, the influence of the sequence in which local and global imperfections are applied on the resulting buckling capacity could also be investigated to better understand interaction effects and their implications for design.

CRediT authorship contribution statement

M. Radwan: Writing – original draft, Visualization, Validation, Software, Resources, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **B. Kövesdi:** Writing – review & editing, Supervision, Resources, Project administration, Methodology, Investigation, Funding acquisition, Formal analysis, Data curation, Conceptualization.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data availability

Data will be made available on request.

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